83

TRIAXIAL TESTING OF OPEN-TYPE BITUMINOUS MIXTURES

JAN. 1958
NO. 4

by

J. C. Oppenlander
&
W. H. Goetz

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

TECHNICAL PAPER

TRIAXIAL TESTING OF OPEN-TYPE BITUMINOUS MIXTURES

TO: K. B. Woods, Director
Joint Highway Research Project

January 30, 1958

FROM: H. L. Michael, Assistant Director

File: 2-4-11
Project: C-36-6K

Attached is a technical paper entitled, "Triaxial Testing of Open-Type Bituminous Mixtures," by J. C. Oppenlander and W. H. Goetz. It has been prepared for presentation at the Annual Meeting of The Association of Asphalt Paving Technologists in Montreal, Canada, February 17-19, 1958.

The report concerns the results of a research study that consisted of carefully testing laboratory-compacted specimens by the open-system triaxial compression test to determine their stability under the influence of different variables. A final report on this study was presented to the Board several months ago.

The paper is presented to the Board for the record and approval to present.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Assistant Director
Joint Highway Research Project

HLM:acc

Attachment

cc: A. K. Branham
J. R. Cooper
W. L. Dolch
W. H. Goetz
J. T. Hallett
F. F. Havey
G. A. Hawkins
G. A. Leonards
J. F. McLaughlin
R. D. Miles
R. E. Mills
B. H. Petty
M. B. Scott
C. E. Vogelgesang
J. L. Waling
J. E. Wilson
E. J. Yoder

TECHNICAL PAPER

TRIAXIAL TESTING OF OPEN-TYPE BITUMINOUS MIXTURES

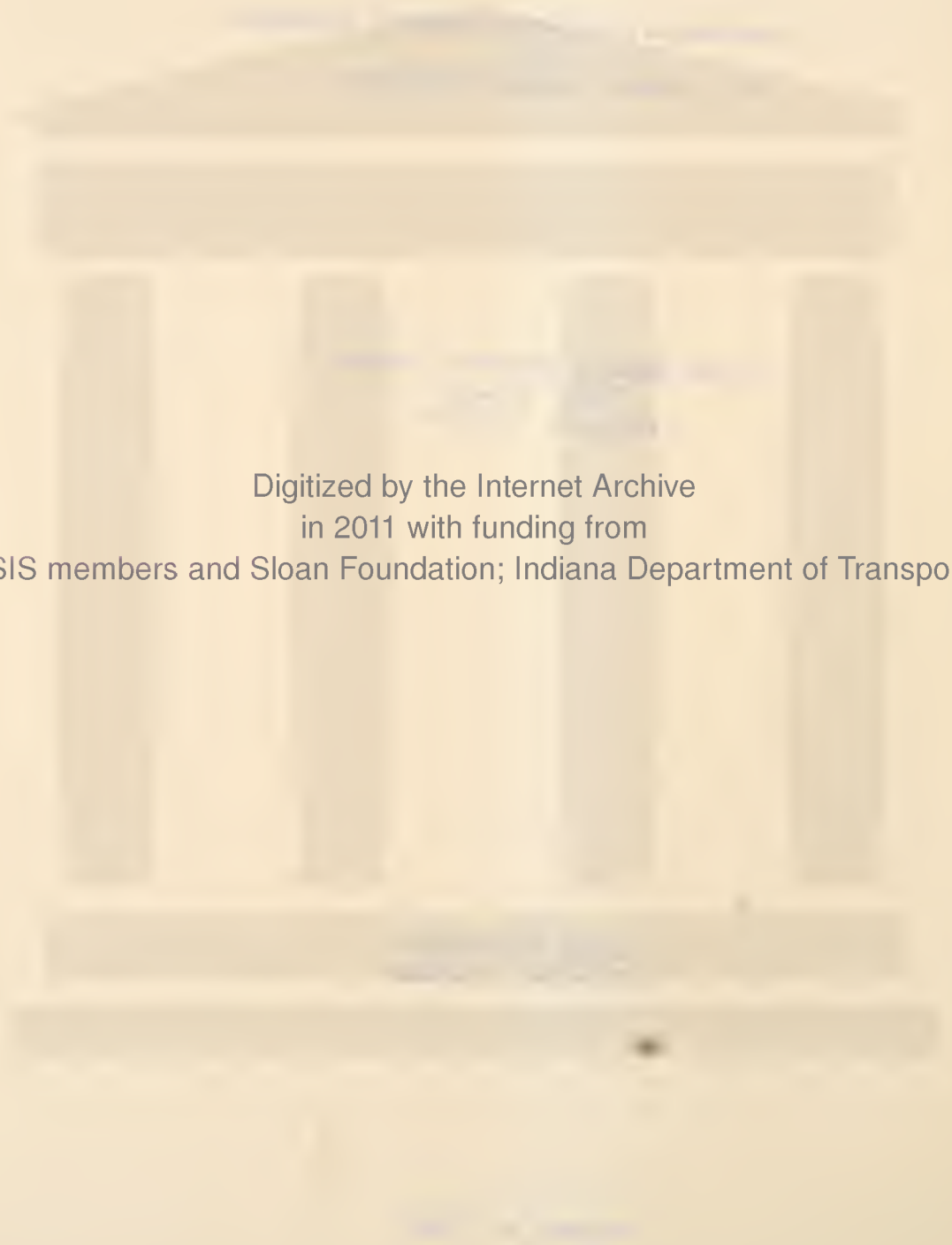
by

J. C. Oppenlander, Formerly Research Assistant
and
W. H. Goetz, Research Engineer

Joint Highway Research Project
File: 2-4-11
Project: C-36-6K

Purdue University
Lafayette, Indiana

January 30, 1958



Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

http://www.archive.org/details/triaxialtestingo5804oppe

SYNOPSIS

From field observation, bituminous mixtures with both one-size and open-graded aggregate gradations appear to perform satisfactorily for many service conditions. The results of conventional triaxial testing, on the other hand, show these one-size mixtures to be unstable under present design criteria, while the open-graded mixtures produce acceptable stability values. This illustrates that the present methods and theories of rational triaxial testing as applied to bituminous mixtures with a one-size aggregate fail to evaluate the true in-service stability properties of these paving mixtures.

This research study consisted primarily of carefully testing laboratory-compacted specimens by the open-system triaxial compression test to determine their stability under the influence of different variables. These were aggregate gradation, confining pressure, specimen height, and degree of compaction. The two aggregate gradations, open-graded and one-size, were incorporated with an asphalt cement into bituminous mixtures. The variable of specimen height necessitated the molding of specimens whose height to diameter ratios were less than two Therefore, this phase of the study was referred to as the irrational triaxial compression test.

The laboratory results were presented in the form of graphical stress diagrams with the confining pressure plotted against the normal stress. The open-graded mixture produced a direct correlation with the linear Coulomb equation, represented by the two constants, (c) cohesion and (ϕ) angle of internal friction. However, different values of cohesion and angle of internal friction were obtained by the rational and the

irrational triaxial tests.

The triaxial testing of the one-size mixture showed that low shearing strength was developed at low values of confining pressure and at small values of specimen deformation. With an increase in the confining pressure and/or the amount of specimen deformation, high values of shearing strength were obtained for this paving mixture. For the range of confining pressures investigated, 0 to 150 psi, approximately 90 percent of the potential shearing resistance was mobilized when the test specimen was deformed to a strain of ten percent. The graphs of confining pressure versus normal stress at various strain values indicated a curvilinear relationship between these two parameters. Thus, the shearing strength of this one-size mixture cannot be ascertained wholly by such constants as the cohesion and the angle of internal friction, variables for this paving mixture. The complex interaction of lateral support, rearrangement of aggregate particles, and change in specimen volume prevented the exact evaluation of the effects of specimen height on the triaxial stability of the one-size mixture.

INTRODUCTION

Many laboratory investigations have been conducted in an attempt to measure the strength or stability of compacted bituminous-aggregate mixtures and to develop empirical methods for the design of these mixtures. However, upon reviewing the many published articles written on these subjects, it is readily apparent that the established design procedures are only applicable to bituminous mixtures possessing a dense-graded aggregate. Since certain bituminous mixtures having a very open aggregate gradation cannot retain a given molded shape without proper support, it is evident that these paving mixtures must develop required confining effects to sustain applied traffic loads. In most of the design tests, except the triaxial test method, the test specimens of these mixtures would possess a very low stability value because the methods do not provide sufficient confinement. Since the triaxial compression test does provide for controlled confinement to the test specimen, it provides a basis for the fundamental evaluation of the true stability of bituminous paving mixtures regardless of gradation.

Since a bituminous-aggregate mixture is a three-phase system with properties not unlike those of a granular soil mass, some bituminous paving engineers have projected that the strength properties of these bituminous-construction materials could be predicted by the same strength theory applicable to soils. This concept has been investigated, and it has been determined that the observed action of a bituminous-aggregate mixture under the action of a loading system is more nearly in accordance with Mohr's theory of strength than with any of the other strength theories based on shear. It may be stated, then, that this analysis provides a basic and logical approach to the evaluation of the strength of bituminous-aggregate mixtures.

The present practice in the evaluation of triaxial test information on bituminous mixtures is predicated on the assumption that the Mohr rupture envelope is a straight line. Thus, constant values of the angle of internal friction and of cohesion are assigned to a given bituminous paving mixture for any given condition. However, certain bituminous mixtures may actually possess a Mohr rupture envelope that is curved.

It would appear, then, that the use of the triaxial test and the interpretation of triaxial test data on the basis of Mohr theory should provide a realistic measure of bituminous mixture strength. However, for the specific case of two mixtures widely used in Indiana as components of bituminous surfacing, it has been demonstrated that, while each performs satisfactorily as a paving mixture for many service conditions, only one of them possesses adequate stability when evaluated by conventional triaxial testing methods.

These two mixtures are made from aggregate gradations that have been referred to as open-graded and one-size. For the purposes of this study, a "one-size" aggregate is a material which passes the 3/4-in. sieve and is retained on the No. 4 sieve. An "open-graded" aggregate is one that is graded from coarse to fine without any material passing the No. 200 sieve.

It was the purpose of this study, then, to investigate the triaxial testing of these "open-type" mixtures in an attempt to learn how they should be tested and/or the test results interpreted in order to provide a realistic measure of stability in the laboratory. In this connection, it was recognized that commonly used confining pressures (up to about 60 psi) may not be great enough for proper testing, that the amount of

deformation that the specimen undergoes in the usual laboratory tests may be insufficient, or that the assumption of a straight-line Mohr rupture envelope may not be valid.

The two aggregate gradations were mixed hot with asphalt cement and formed into triaxial test specimens by means of double-plunger compaction, both with and without vibration in order to produce specimens at different density levels. Both rational and irrational triaxial test specimens were formed. A rational triaxial test is described as one in which the diameter of the test specimen is at least four times the maximum aggregate size, and the height of the specimen is at least two times its diameter. If the height to diameter ratio of the specimen is less than two, the triaxial compression test is defined as an irrational or modified triaxial test. The specimens were tested at confining pressures up to 150 psi and with strain values up to 20 percent.

MATERIALS

The mineral aggregate selected for this research project was a commercial crushed limestone, the physical characteristics of which were not altered in the laboratory except for gradation. This crushed-limestone aggregate had a bulk specific gravity of 2.66, and apparent specific gravity of 2.71, and an absorption of 0.74 percent.

This aggregate was used at two gradations, designated as open-graded and one-size. The open-graded aggregate corresponded to the requirements for the Hot Asphaltic Concrete Binder Course, and the one-size aggregate followed the Bituminous Coated Aggregate Surface (size No. 9 aggregate) gradation, as presented in the Indiana State Highway Department Specifications (17). Samples of these two gradations are pictured in Fig. 1. The sieve analyses of these open-graded and one-size aggregates are presented in Table 1. A graphical plot of the gradations of these aggregates is shown by the aggregate gradation curves in Fig. 2.

Fig. 1 Crushed Limestone Aggregate, Open-Graded and One-Size Gradations

TABLE 1

Sieve Analysis of Open-Graded and One-Size Aggregates

(Percent by Weight)

Sieve		Grading	
Passing	Retained	Open	One-Size
--	3/4"	0	0
3/4"	1/2"	17.5	29.2
1/2"	3/8"	21.4	35.4
3/8"	#4	21.4	35.4
#4	#6	2.9	0
#6	#8	1.7	0
#8	#16	10.5	0
#16	#50	20.7	0
#50	#100	3.6	0
#100	#200	0.3	0
#200	--	0	0

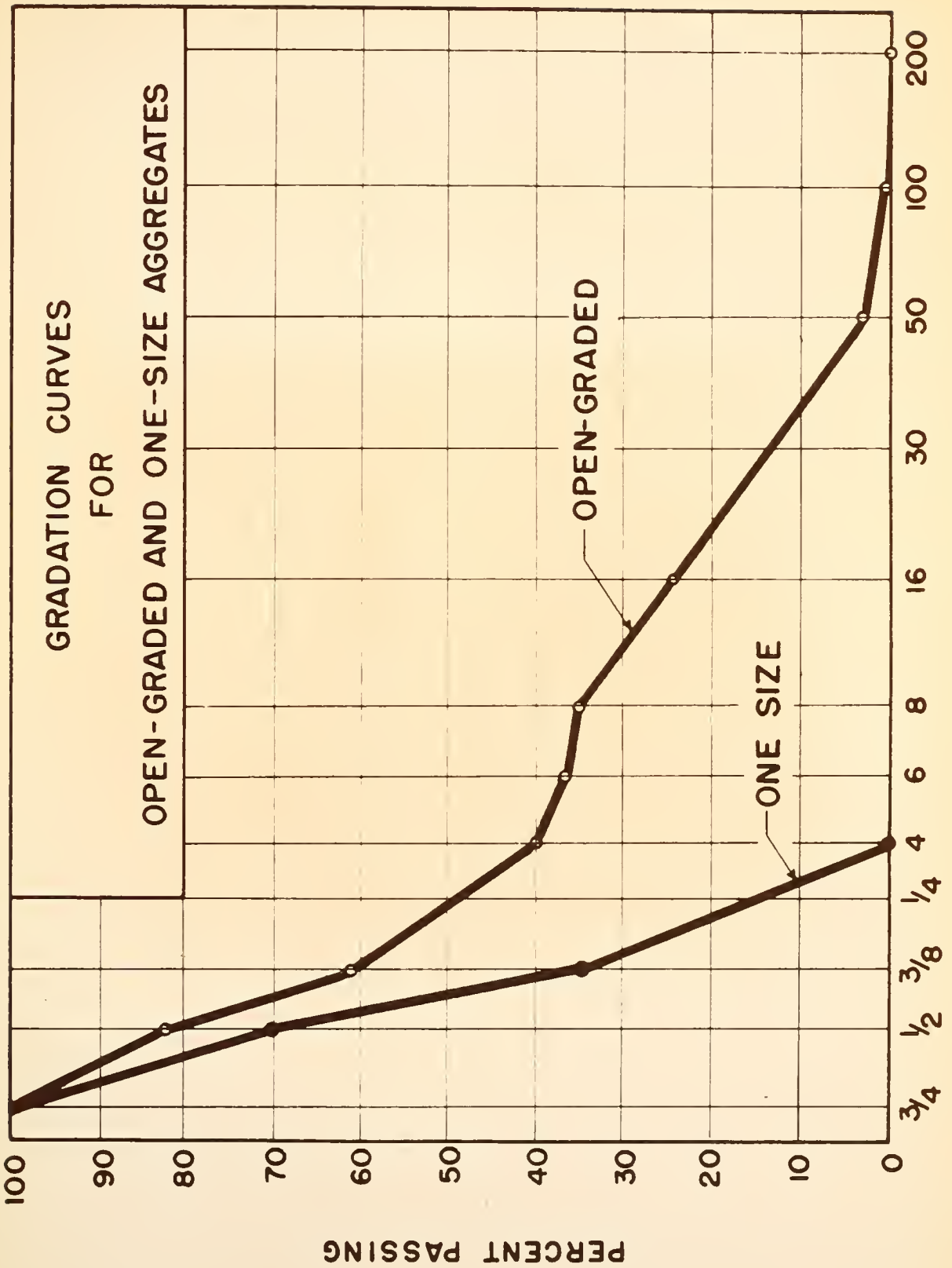

GRADATION CURVES
FOR
OPEN-GRADED AND ONE-SIZE AGGREGATES
OPEN-GRADED
ONE SIZE
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
3/4
1/2
3/8
1/4
4
6
8
16
30
50
100
200
SIEVE SIZE (LOG SCALE)

FIG. 2

The bituminous material used in this study was an asphalt cement classified as a 60-70 penetration grade. Several standard tests were performed in the laboratory on this asphaltic material. The results of these tests along with the designation of the applicable American Society for Testing Materials standard method of test are presented in Table 2.

TABLE 2

Physical Tests on 60-70 Penetration Grade Asphalt Cement

Test	ASTM Test No.	Results
Penetration - 1/100 cm (77 F, 100 g, 5 sec)	D5-52	66
Specific gravity (77 F / 77F)	D70-52	1.018
Ductility - cm (77 F, 5 cm/min)	D113-44	150+
Solubility in CCl_4 - %	D165-42	99.81
Loss on heating - % (50 g, 5 hr., 325F)	D6-39T	0.01

The asphalt content for the two aggregate gradings was maintained at 5.0 percent by weight of the total mixture throughout this research investigation. This percentage of asphalt cement used in the open-graded and the one-size aggregate mixtures is that normally specified for actual construction by the Indiana State Highway Department.

PROCEDURE

In this study the mineral aggregates were separated into various sieve size fractions and then were recombined in the desired proportions just prior to the preparation of the test specimens. This procedure provided a high degree of control over aggregate gradation. All of the specimens tested were four inches in diameter. The specimens for the rational triaxial test were ten inches high and those for the irrational test were four inches high.

Preparation of the Test Specimen

The following method was utilized in the preparation of the test specimens of the open-graded type. After heating the prepared aggregate and the asphalt cement to the temperatures of 300 $\pm$ 10F, and 275 $\pm$ 10F, respectively, these two components of a bituminous mixture were thoroughly combined in a heated bowl with a Hobart mixer for a period of two minutes. Then this bituminous-aggregate mixture was placed in a heated compaction mold in equal layers. Each layer, four layers for the rational test specimens and two layers for the irrational test specimens, was rodded 25 times with a 3/4-in. round steel bar that weighed 1.4 lb. The triaxial test specimen was compacted by the double-plunger method with a pressure of approximately 2170 psi applied to each end of the specimen for one minute. Then the compaction mold containing the compacted specimen was immersed for a period of 8 to 12 minutes in a circulating cold-water bath. After the cooling period, the specimen was removed by disassembling the compaction mold. These specimens, illustrated in Fig. 3, were stored at room temperature until the time for testing.

Fig. 3 Open-Graded and One-Size Mixture Specimens and One-Size Specimen Container

With some exceptions, the preparation of the one-size mixture specimens was performed in the same manner as for specimens of the open-graded mixture. The following deviations from the above procedures were used. The temperature of the heated aggregate and asphalt were maintained at 200 $\pm$ 10F and 235 $\pm$ 10F, respectively, in advance of the mixing operation. In the compaction of this bituminous mixture by the double-plunger procedure, a pressure of approximately 430 psi was applied to each end of the specimen. This modification minimized aggregate breakage. To provide better compaction, the compaction mold was vibrated by striking it with a leather mallet for 50 blows, while the compacting pressure was maintained. A light-weight, sheet-metal container was placed around the test specimen after it was removed from the cooled compaction mold. These containers were not removed until the specimens were to be tested. A one-size mixture specimen and a sheet-metal container are pictured in Fig. 3.

In one phase of this study, one-size mixture specimens with density values greater than that produced by the normal procedures were desired. To produce these, each layer, after rodding with the steel bar, was further compacted by placing the compaction foot of a pneumatic vibrator around the surface of the mixture layer for one minute, with the pneumatic vibrator operating at a pressure of 60 psi. This pneumatic vibrator with the compaction foot, as developed by the authors for this research investigation, is illustrated in Fig. 4.

Fig. 4 Pneumatic Vibrator With Compaction Foot and Specimen Mold

Density Determinations

During the triaxial compression test of the open-graded specimens, there was no measurable evidence of any change in the volume of the sample. Therefore, it was assumed that the volume of the specimen remained constant throughout the triaxial test. Before the actual testing, each open-graded specimen was weighed to the nearest 0.1 g on a torsion balance. The height of the specimen was determined to the nearest 0.01 in. by averaging six measured heights obtained from reading a referenced Ames dial indicator mounted on a support stand. These values permitted the density of the specimen to be computed.

However, the specimens of the one-size mixture experienced a measurable reduction in volume when they were subjected to the triaxial compression test. Since a measure of this volume change was required for the proper reduction of the triaxial test data, the volumes (and also the densities) of the specimen both before and after the test were determined. The accepted procedure (20) of coating the specimen with melted paraffin for the density evaluation was not feasible in this case as the paraffin flowed into the voids of the mixture and produced erroneous results. Therefore, the following procedures were adopted to measure the densities and the volumes of the one-size mixture specimens with an acceptable degree of accuracy.

1. The weight in air of the test specimen was determined on the torsion balance. (All weights were obtained to the nearest 0.1 g.)
2. The height of the specimen was determined to the nearest 0.01 in. by averaging six measured heights obtained from reading the referenced dial indicator mounted on a support stand.

3. The cylindrical surface of the test specimen then was tightly wrapped in a thin sheet of polyethylene plastic, and the weight in air of the specimen plus the plastic sheeting was recorded.

4. After the ends of the specimen and the plastic seams were sealed by brushing with melted paraffin so as to completely water-proof the specimen, the weight in air of the encased specimen was determined.

5. After taring the torsion balance for the suspension rod and the suspension basket which was completely submerged in the water contained in a 15-gal. can, the encased specimen was placed in the suspension basket, and the weight of the specimen in water at room temperature was obtained. The laboratory setup for the density determination is pictured in Fig. 5.

After determining these four weights both before and after the triaxial compression test, and knowing the specific gravity values of the polyethylene plastic and the paraffin, the volumes and the densities of the specimen before and after the triaxial test and the change in volume were calculated.

Triaxial Compression Test

The shearing strengths of these bituminous-aggregate mixtures were ascertained by the triaxial compression test of the open-system type by using compressed nitrogen to provide confining pressure. These triaxial tests were performed on identical specimens for each series of

Fig. 5 Equipment for Specimen Density Determination

investigations. For a range of confining pressures (0 to 150 psi at 30 psi intervals), the test specimens were deformed at a uniform rate of 0.05 in. per min. within a temperature range of 77 ± 5F. The complete assembly for the triaxial compression test is shown in general views by Fig. 6 for the rational test and in Fig. 7 for the irrational test.

Data Reduction

In the triaxial stability analysis of the open-graded test specimens, the volume of the specimen was assumed to remain constant as there was no measurable evidence of any change in volume during the triaxial test. Based on this assumption, the expression for the corrected cross-sectional area of the specimen experiencing no volume change is derived in Appendix A. An example of a plot of the corrected cross-sectional area of the specimen as a function of the change in the specimen height is presented in Fig. 16 (see Appendix A) for specimens with an initial cross sectional area of 12.57 sq. in. and an initial height of 10.25 in.

During the triaxial compression tests on the laboratory compacted specimens of the one-size mixture, the volume of the specimen was reduced a measurable amount, thus increasing the density of the specimen as the test progressed. The expression for the corrected cross-sectional area of the specimen experiencing a volume reduction is derived in Appendix B upon the assumption that the reduction in volume is directly proportional to the change in the specimen height. As was the case for specimens of the open-graded mixture, the average heights of the various groupings of the test specimens were computed since the corrected area was dependent upon the initial specimen height. By dividing the total change in the

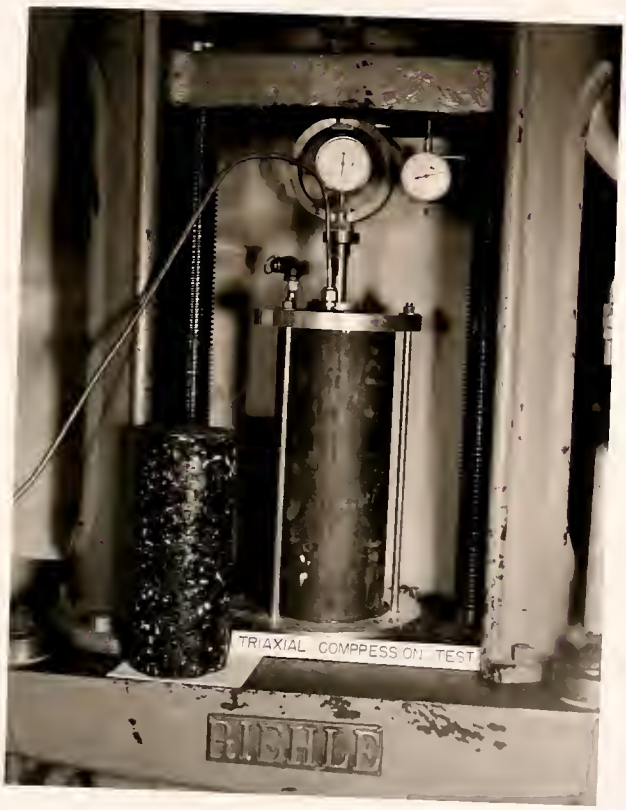


Fig. 6 Rational Triaxial Compression Test

Fig. 7 Irrational Triaxial Compression Test

specimen volume in cubic inches by the total change in the specimen height in inches, the constant "C" in $in.^3$ per in. was evaluated for each test specimen. As this relative measure of the degree of volume change varied over a limited range for the individual specimens of a given grouping, a parameter of curves depending on different "C" values was plotted for each of the average initial-specimen-height groups. An example of one set of these curves which provide the corrected cross-sectional area of the specimen as a function of the change in the specimen height is illustrated in Fig. 17 (Appendix B).

A stress-strain diagram was prepared for each triaxial test. The total normal stress for various percent strains and the peak condition (if it existed) were obtained from these curves.

A convenient method for analyzing triaxial test results, without plotting the Mohr diagram, is to plot a graph of the confining pressure versus the total normal pressure. This plot will yield a straight line if the Mohr rupture envelope is linear. W. M. Aldous, R. C. Herner, and M. H. Price (1) have derived expressions for determining the angle of internal friction and the cohesion from the slope and the vertical intercept of this graph of the lateral and normal stresses. Although the utilization of these two equations are only applicable to isotropic bituminous mixtures having a linear Mohr rupture envelope (10), the plot of the confining pressure versus the total normal stress provides a convenient, graphical analysis of the strength properties of non-isotropic bituminous-aggregate mixtures (6) having curved Mohr rupture envelopes. For these cases, the plots will be curved, and values of the angle of internal friction and cohesion will vary for different confining pressures.

RESULTS

In this laboratory investigation, triaxial stability values were determined for compacted bituminous mixtures by the open-system triaxial compression test using confining pressures up to 150 psi. The variable conditions introduced were aggregate gradation, confining pressure, specimen height, and degree of compaction.

The test data developed in this study are presented in tabular form in Appendix C which includes Tables 4, 5, 6, 7, 8, and 9. In addition to strength data used to plot various curves presented in the figures included in this section of the report, the tables include density data as a matter of record.

It was found that the stress-strain curves, representing the two bituminous mixtures, open-graded and one-size, were considerably different. As shown in Fig. 8, the stress-strain diagrams for the open-graded mixture reached a peak normal stress value at a relatively low strain value, while the normal stress continued to increase at a decreasing rate with an increase in the strain value for the one-size mixture. For this reason, it is appropriate to consider the test results for these two mixtures separately.

Open-Graded Mixture

The shearing resistance of the open-graded paving mixture is illustrated in Fig. 9 where the confining pressure versus the peak normal stress was plotted from stress values ascertained from rational triaxial tests with confining pressures ranging from 0 to 150 psi, at 30 psi intervals. Crushed limestone was used in this mixture at an asphalt content of 5.0 percent by weight of the total mixture.

TYPICAL STRESS-STRAIN CURVES

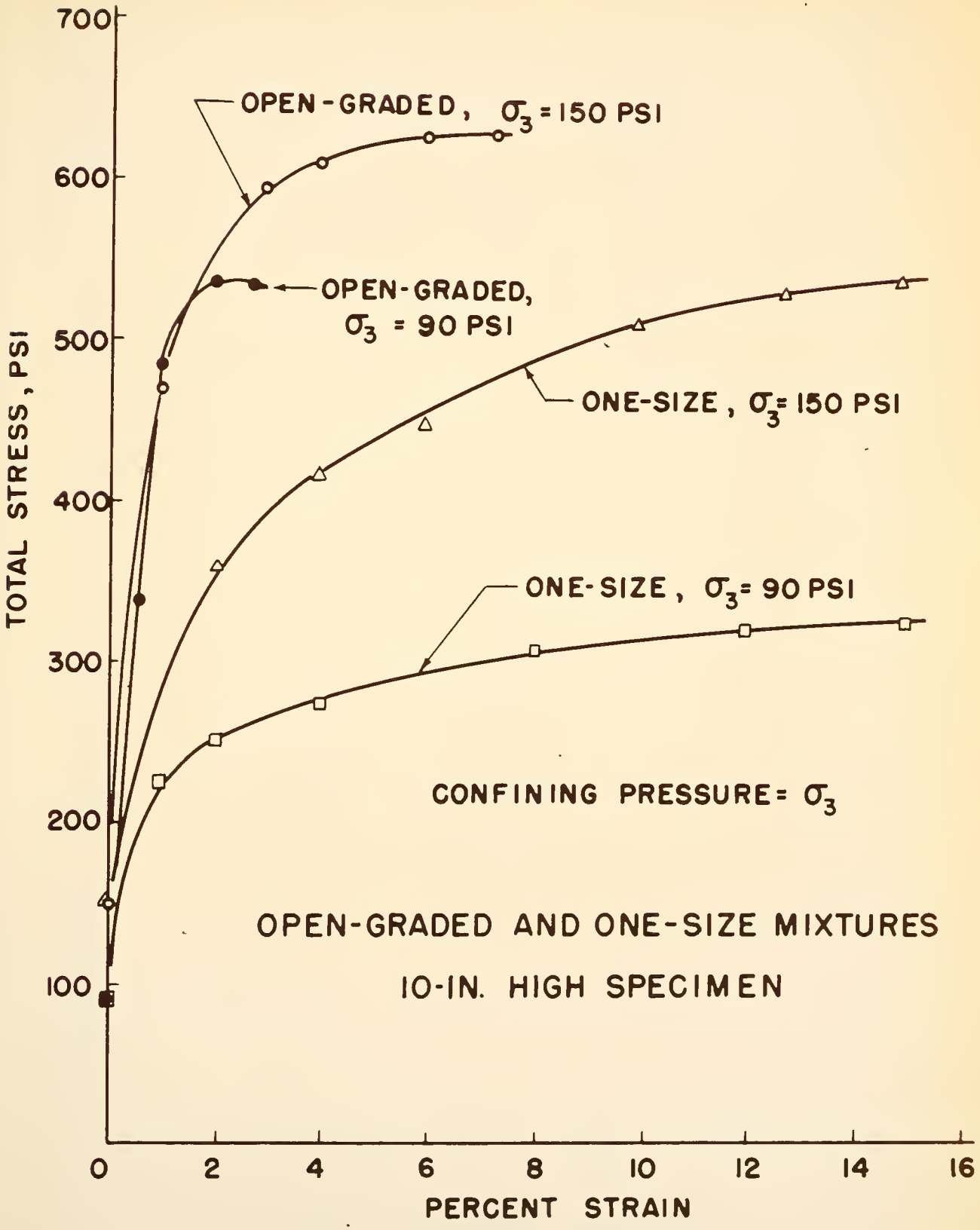


FIG. 8

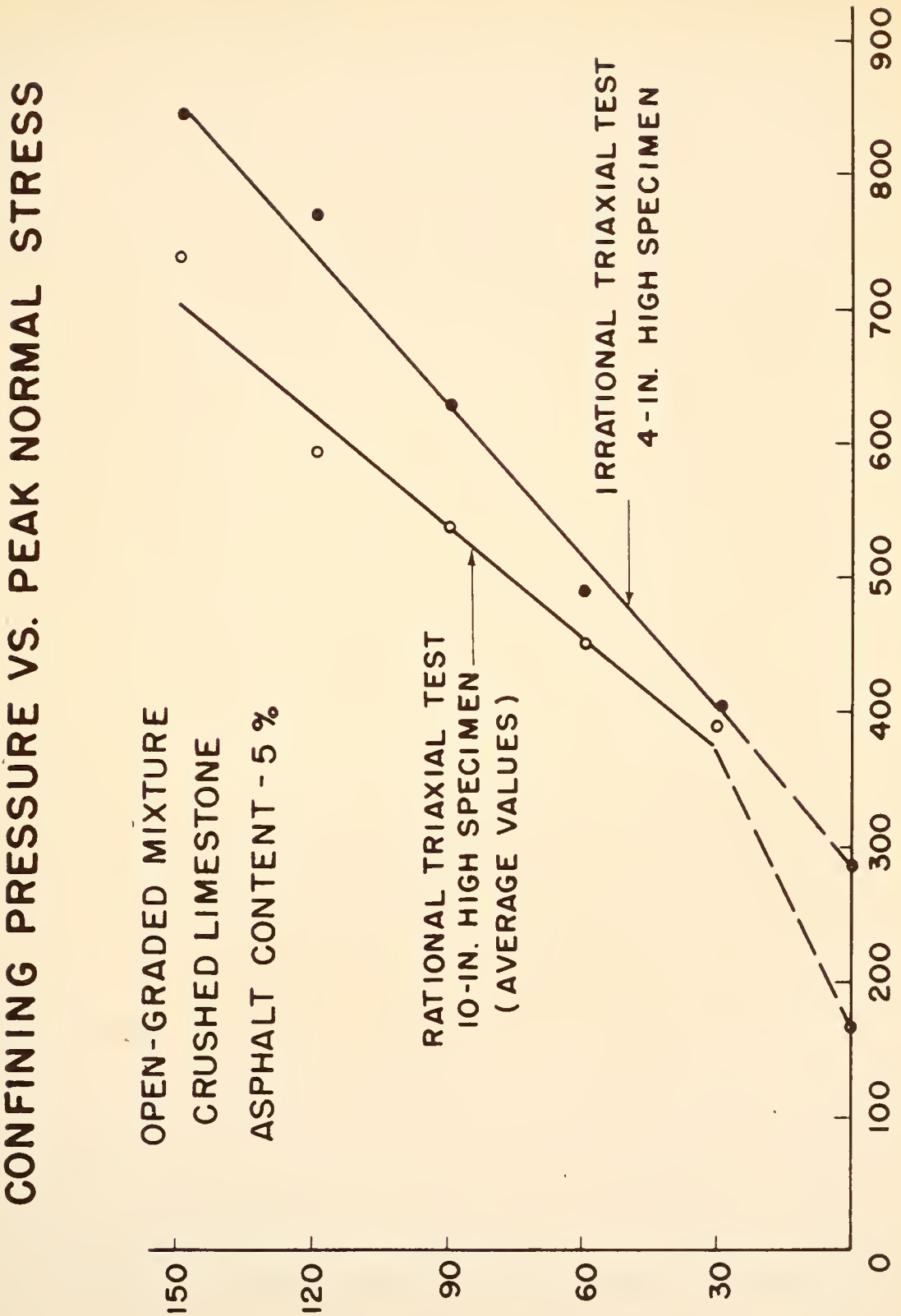

CONFINING PRESSURE VS. PEAK NORMAL STRESS
OPEN-GRADED MIXTURE
CRUSHED LIMESTONE
ASPHALT CONTENT - 5 %
RATIONAL TRIAXIAL TEST
10-IN. HIGH SPECIMEN
(AVERAGE VALUES)
IRRATIONAL TRIAXIAL TEST
4-IN. HIGH SPECIMEN
CONFINING PRESSURE - PSI
150
120
90
60
30
0
100
200
300
400
500
600
700
800
900
PEAK NORMAL STRESS - PSI

FIG. 9

Rational Triaxial Test. The open-graded bituminous-aggregate mixture possessed appreciable triaxial stability, 390 psi, at the low confining pressure of 30 psi. The shearing strength increased linearly with confining pressure until a value in excess of 700 psi was reached at the 150-psi level. Since this linear relationship between confining pressure and peak normal stress is analogous to a straight-line Mohr rupture envelope (1), the shearing strength of this bituminous mixture may be ascertained by the Coulomb equation and represented by the constant strength properties, cohesion (c), and angle of internal friction (ϕ). However, the peak normal stress at 0 psi confining pressure did not agree with the linear relationship, thus indicating that the Mohr rupture envelope is curvilinear in the range of low confining pressures (less than 30 psi). This condition has been validated by previous rational triaxial test investigations in both the soil mechanics and the bituminous-mixture areas.

Specimen Height (Irrational Triaxial Test). The results obtained in evaluating the effects of specimen height on the triaxial stability of the open-graded mixture also are presented in Fig. 9. The relationship between the plotted stress values was best represented by a straight line. These irrational triaxial tests were performed on compacted bituminous specimens having the same type of aggregate and the same asphalt content as the specimens used in the previously discussed rational triaxial test, except that 4-in. high specimens were used (height to diameter ratio was less than two). For corresponding confining pressures, the peak normal stresses obtained in the irrational

triaxial tests were greater than those stresses determined in the rational triaxial testing.

These greater stress values were produced by the additional lateral support afforded the irrational test specimen by the shearing stresses developed between the ends of the specimen and the upper and the lower loading plates. This reasoning also explains the close position of the peak normal stress for the 0 psi confining pressure to the extension of the straight line representing the irrational triaxial tests in Fig. 9.

For both the rational and the irrational triaxial tests of the open-graded mixture, the peak normal stresses were reached at strain values of less than six percent. The strain values at which the peak stability values occurred centered around one percent for the 30 psi confining pressure, and they gradually increased with an increase in the confining pressure to the 150-psi level where the strain values were over five percent (see Fig. 8).

Although the two series of test values plotted in Fig. 9 appear to be characterized by a linear relationship, the slopes of these two straight lines are different. This indicates that the rational and the irrational triaxial tests do not produce equal measures of the strength properties, cohesion and angle of internal friction, of this open-graded mixture.

One-Size Mixture

The other aggregate gradation investigated in this study of triaxial compression testing was the one-size mixture. This paving

mixture has performed more satisfactorily under heavy traffic than would be predicted from laboratory strength tests.

Rational Triaxial Test. The shearing strength of this one-size mixture is illustrated in Fig. 10 where the confining pressure versus the normal stress at various values of percent strain was plotted. These stress values were determined by rational triaxial tests with confining pressures ranging from 0 to 150 psi, at 30 psi intervals, on compacted crushed-limestone specimens having an asphalt content of 5.0 percent by weight of the total mixture. As evidenced by the typical stress-strain curves for this one-size mixture at various confining pressures (see Fig. 8), the normal stresses increased at a decreasing rate with an increase in the amount of strain applied to the test specimen. However, this decreasing rate was found to vary inversely with the confining pressure. This is illustrated by comparing the normal stresses at 1.0 and 15.0 percent strain in Fig. 10 for the two confining pressures of 30 and 150 psi. Thus, at the 30 psi level shearing strengths of approximately 100 and 160 psi were developed at 1.0 and 15.0 percent strain, respectively; while the corresponding shear values at a confining pressure of 150 psi were 300 and 540 psi.

The absence of any peak normal stress for these one-size mixtures prompted the analysis of the triaxial strength data at 1.0, 5.0, 10.0, and 15.0 percent strain. The physical limitations of the triaxial test apparatus and the difficulty of forming completely homogeneous test specimens of the one-size mixture prevented the determination of normal stress at strain values greater than 15 percent. The value of 15.0 percent strain may also be considered the maximum practical deformation limit for flexible pavement design applications.

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

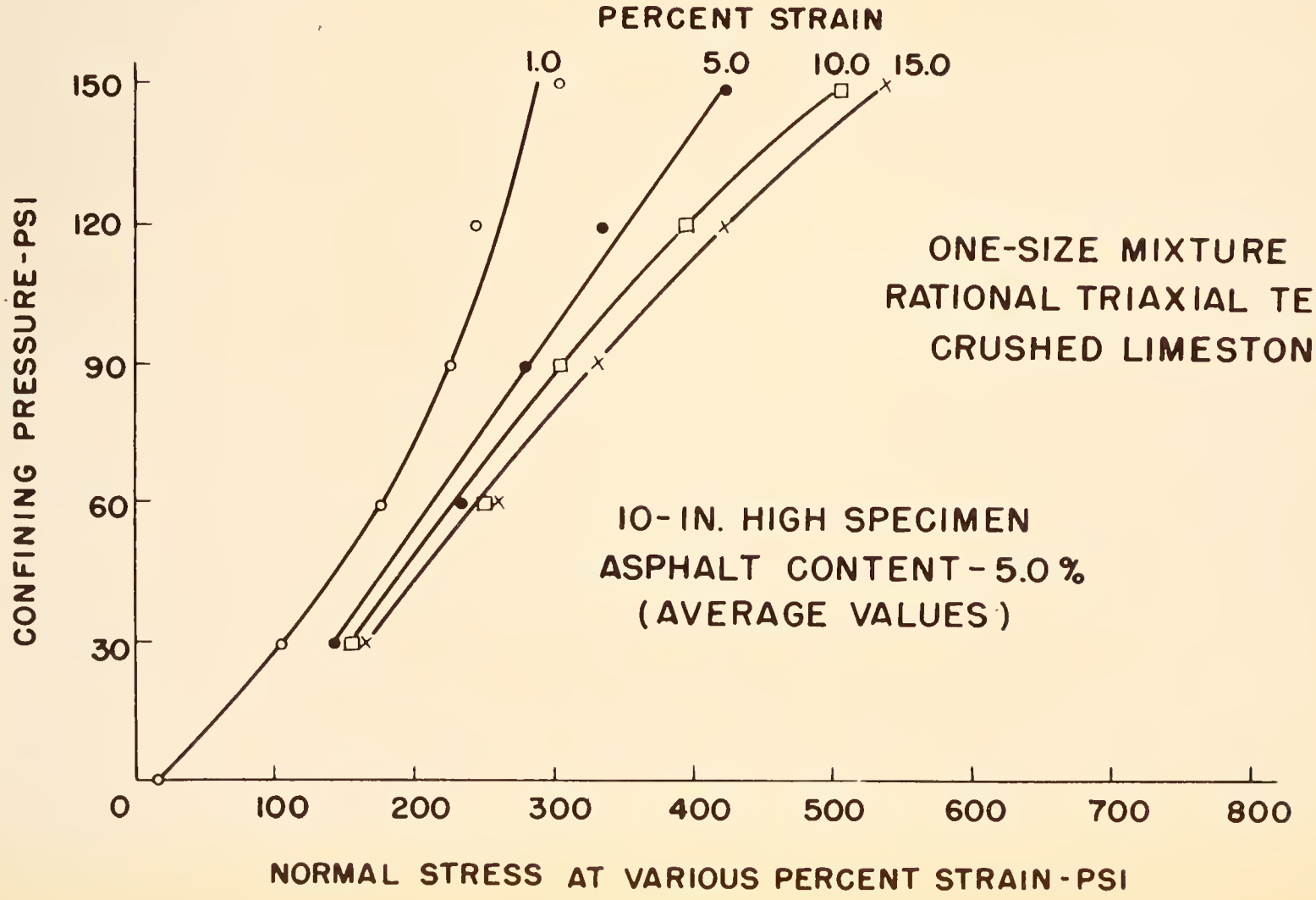


FIG. 10

The relationship between the confining pressure and normal stress at 1.0, 5.0, 10.0, and 15.0 percent strain for the one-size mixture (Fig. 10) is considerably different than that presented for the open-graded mixture (Fig. 9). The normal stresses at 1.0 percent strain increase at a decreasing rate with an increase in the confining pressure. At the 5.0 percent strain value, the graph of these two stress values is best represented by a straight line. The normal stresses at the larger strains, 10.0 and 15.0 percent, follow a curvilinear pattern in which the normal stresses increase at an increasing rate for an increase in the confining pressure.

The variable nature of these stress diagrams indicates that the cohesion and the angle of internal friction were not constants in this one-size mixture when the stability was ascertained by the rational triaxial compression test. The angle of internal friction and/or the cohesion varied throughout the range of confining pressures and the range of strain values. Thus, the Coulomb equation and fixed values of cohesion and angle of internal friction cannot truly represent the triaxial stability of this one-size mixture. Since the triaxial compression test on the one-size limestone mixture produced a peak normal stress near 1.0 percent strain for the 0 psi confining pressure condition, the other three sets of curves must show a reversal in slope in the range of low confining pressures and approach zero at 0 psi confining pressure.

Density Determination. In all the triaxial compression tests (except at 0 psi confining pressure) of the one-size mixture, a measurable reduction in the volume of the specimen occurred. To properly evaluate the triaxial test data, this rearrangement of the aggregate

particles and the reduction in the volume were considered for the one-size specimens by determining their bulk densities both before and after the triaxial compression test.

The bulk density of a compacted-bituminous-mixture specimen is determined by computing the ratio of its weight in air to its bulk volume. The Asphalt Institute (20) has summarized the determination of the bulk volume of the specimen in the following procedures:

1. The bulk volume of a specimen having a dense, impermeable surface is determined as the difference between its weight in air and its weight in water.
2. The bulk volume of a specimen having an open, permeable surface is determined as the difference between its weight in air and its weight in water when coated with paraffin.
3. The bulk volume of a specimen having a dense, smooth surface and formed to dimensions that can be accurately measured is computed from the diameter and the height measurements of the specimen.

The last procedure was valid for the determination of the bulk density of the open-graded specimens. However, none of the three available methods were applicable to evaluate the changes in volume and density that the one-size specimens experienced when they were subjected to a triaxial compression test. As the compacted bituminous-aggregate specimen was changed from a uniform cylindrical shape to a distorted shape as a result of the triaxial test, the method utilizing the measured dimensions was of no practical value. The very porous nature of the one-size mixture necessitated that the specimen be coated in some manner, thus eliminating the first procedure.

The method of coating the compacted specimen with paraffin was attempted, but this proved unsatisfactory as the melted paraffin flowed into the void space. This condition produced erroneous results as the density values were increased because the voids were decreased. Therefore, it was mandatory that a new method of density determination be developed. This procedure entailed the wrapping of the specimens in a carefully-cut thin sheet of polyethylene plastic and then sealing the seam and the ends of the specimen with paraffin. A detailed description of this method is presented under the heading Density Determinations in the Procedure section.

Failure Criterion. It logically may be reasoned that the one-size mixture develops increasing shearing strength with increases in the confining pressure and with increases in the degree of specimen deformation. In order to establish a logical failure criterion for this paving mixture, the rate at which the shearing resistance was mobilized was investigated. This shearing stress development is pictured in Fig. 11. The curve for each confining pressure appears to asymptotically approach a maximum normal stress. Since the normal stresses at 15.0 percent strain are reasonably close in each case to these maximum values, it was arbitrarily assumed that the normal stresses at 15.0 percent strain represented the condition of maximum shearing resistance.

The ratios of the normal stresses at 1.0, 5.0, and 10.0 percent strain to the normal stresses at 15.0 percent strain for the range of confining pressures investigated were calculated and are tabulated in Table 3. The graphical form of these data is presented in Fig. 12, where it may be deduced that at least 92 percent of the shearing strength was mobilized when the bituminous-aggregate mixture had experienced a strain of 10.0 percent. For the purposes of this

NORMAL STRESS VS. PERCENT STRAIN

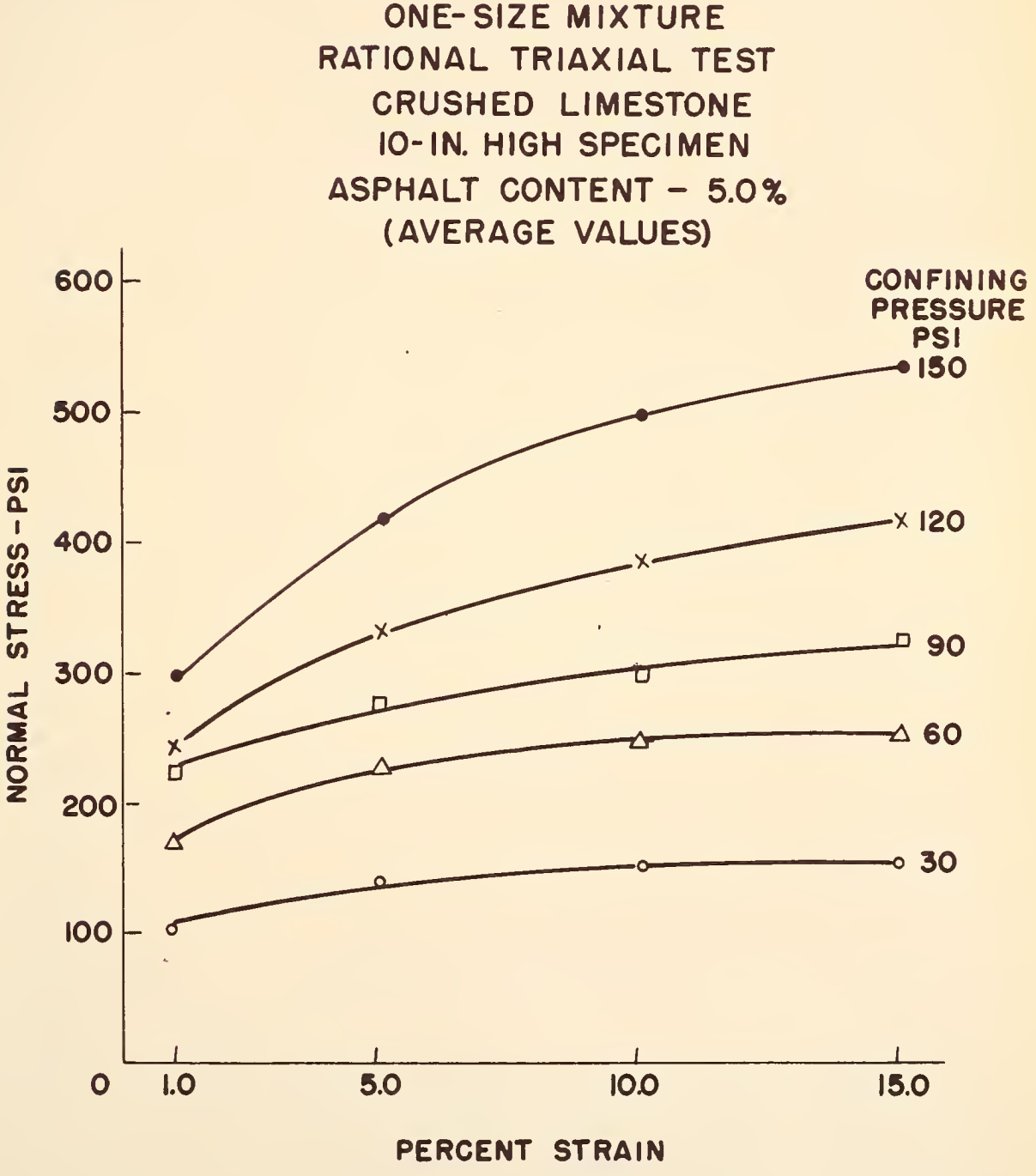


FIG. 11

STRESS RATIO VS. CONFINING PRESSURE

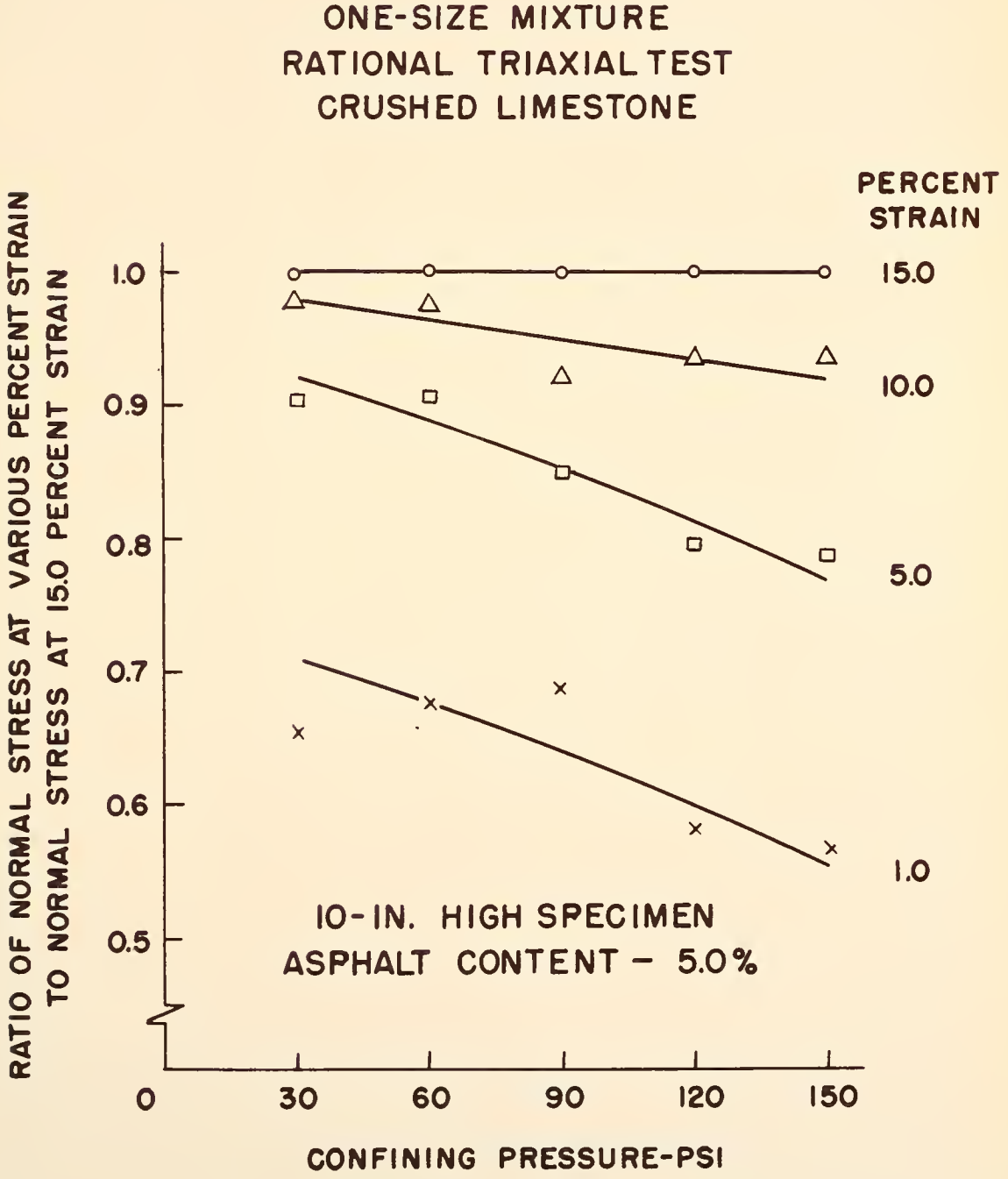


FIG. 12

laboratory investigation, the values of normal stress developed by the one-size mixture at 10.0 percent strain were considered as the practical values of shearing resistance for this paving mixture. This represents a logical compromise between the two important considerations, strength and deformation, for plastic materials.

Degree of Compaction and Specimen Height (Irrational Triaxial Test). The effects of the degree of compaction on the triaxial stability of the one-size mixture were investigated for crushed-limestone specimens having an asphalt content of 5.0 percent by weight of the total mixture. The normal method of forming specimens was by the double-plunger compaction procedure. To obtain one-size specimens with density values greater than those produced by the regular double-plunger compaction method, this compaction procedure was modified to include vibratory compaction of the specimen with a specially designed compaction foot attached to a pneumatic vibrator. The double-plunger method and the double-plunger with vibration method are discussed under the heading Preparation of the Test Specimen in the section Procedure. This vibrator with the compaction foot attached is pictured in Fig. 4.

The results of this rational triaxial testing of the vibrated one-size mixture specimens are shown in Fig. 13 where the confining pressure is plotted against the normal stress at various strain values. These curves of the one-size mixture compacted to a greater density are to be compared to the rational triaxial stability values obtained for the specimens formed at a lower density (see Fig. 10). In each case, the curves followed the same pattern. The normal stresses at 1.0 percent strain increased at a decreasing rate (although the decreasing rate for the vibrated specimens was less than that for the non-vibrated specimens) with an increase in the confining pressure, while the normal stress values

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

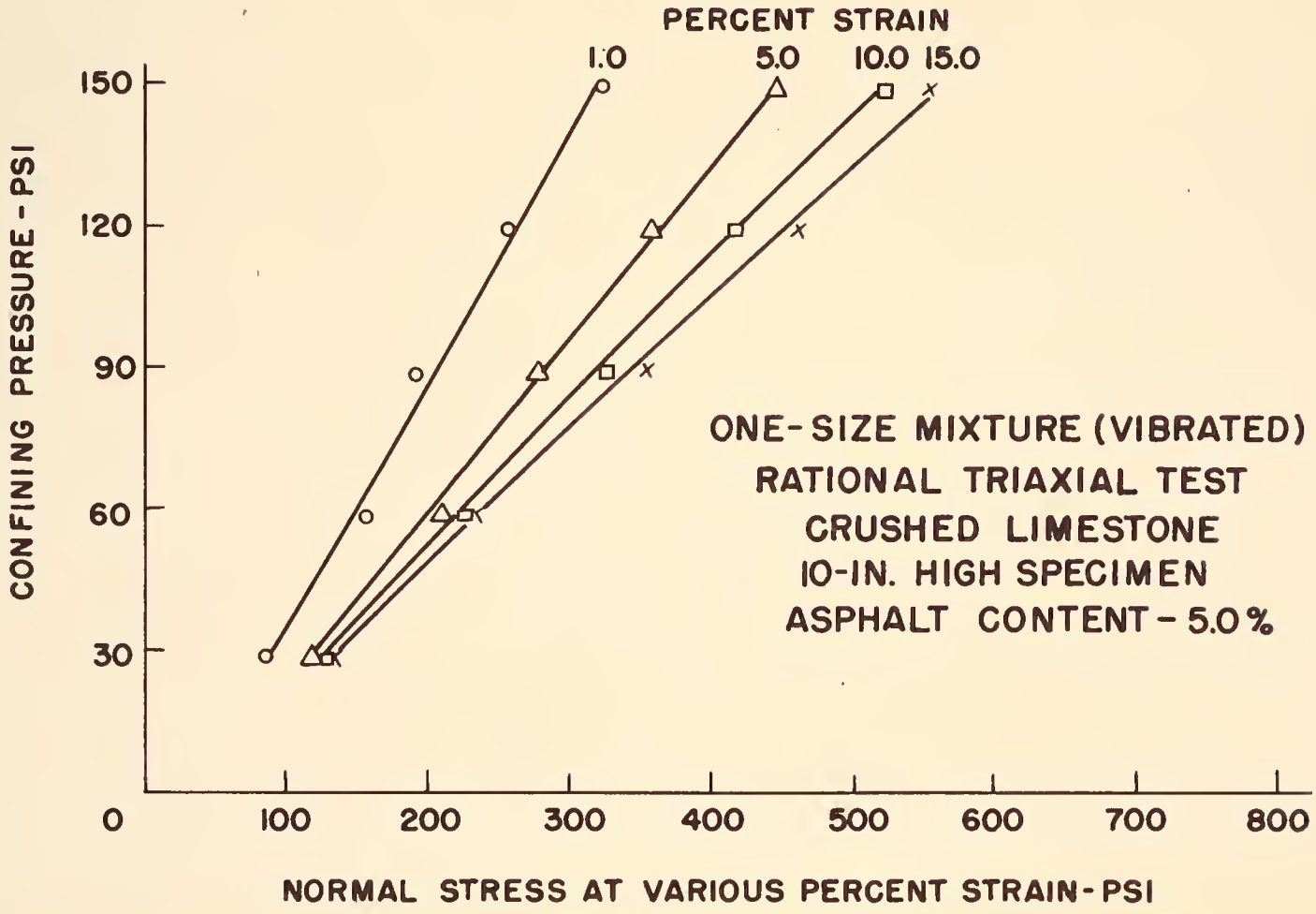


FIG. 13

at 5.0 percent strain showed a linear relationship with confining pressure. The values of normal stress at 10.0 and 15.0 percent strain increased at an increasing rate as higher confining pressures were applied to the rational triaxial specimens. The curvature of the lines representing these relationships is not great, and for many purposes they could be considered as straight. However, strength analysis of this one-size mixture on the basis of an assumed straight Mohr envelope is not valid, as was shown by further testing.

One apparent discrepancy was evident when a comparison of the non-vibrated specimen data (Fig. 10) was made with the vibrated specimen results (Fig. 13). The normal stresses at 30 and 60 psi confining pressures were higher for the non-vibrated specimens (lower densities) than for the vibrated specimens (higher densities), while the reverse was apparent for the shearing stresses at the 90, 120, and 150 psi confining pressures. The probable explanation for this heterodox condition is the interrelated and complicated interaction between confining stresses, rearrangement of the aggregate particles, and reduction in the volume of the triaxial test specimen during test.

The effects of specimen height on the triaxial strength of the one-size mixture were investigated for both non-vibrated and vibrated specimens. The irrational triaxial test results for 4-in. high specimens are presented graphically in Fig. 14 for the double-plunger compaction method and in Fig. 15 for the double-plunger with vibration compaction procedure. The asphalt content was again maintained at 5.0 percent by weight of the total mixture.

The pattern of curves representing the relationship between the confining pressure and the normal stress for the various values of

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

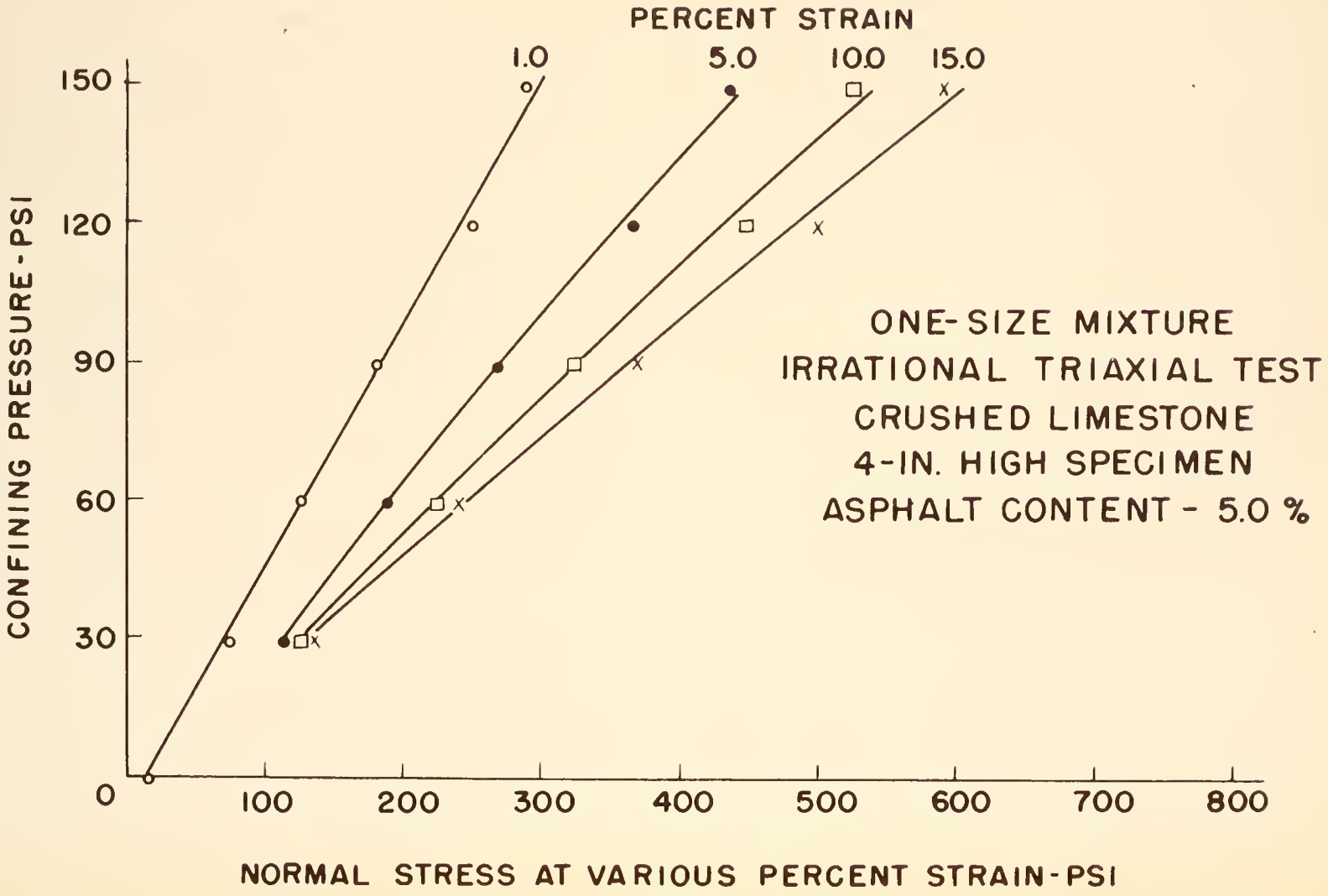


FIG. 14

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

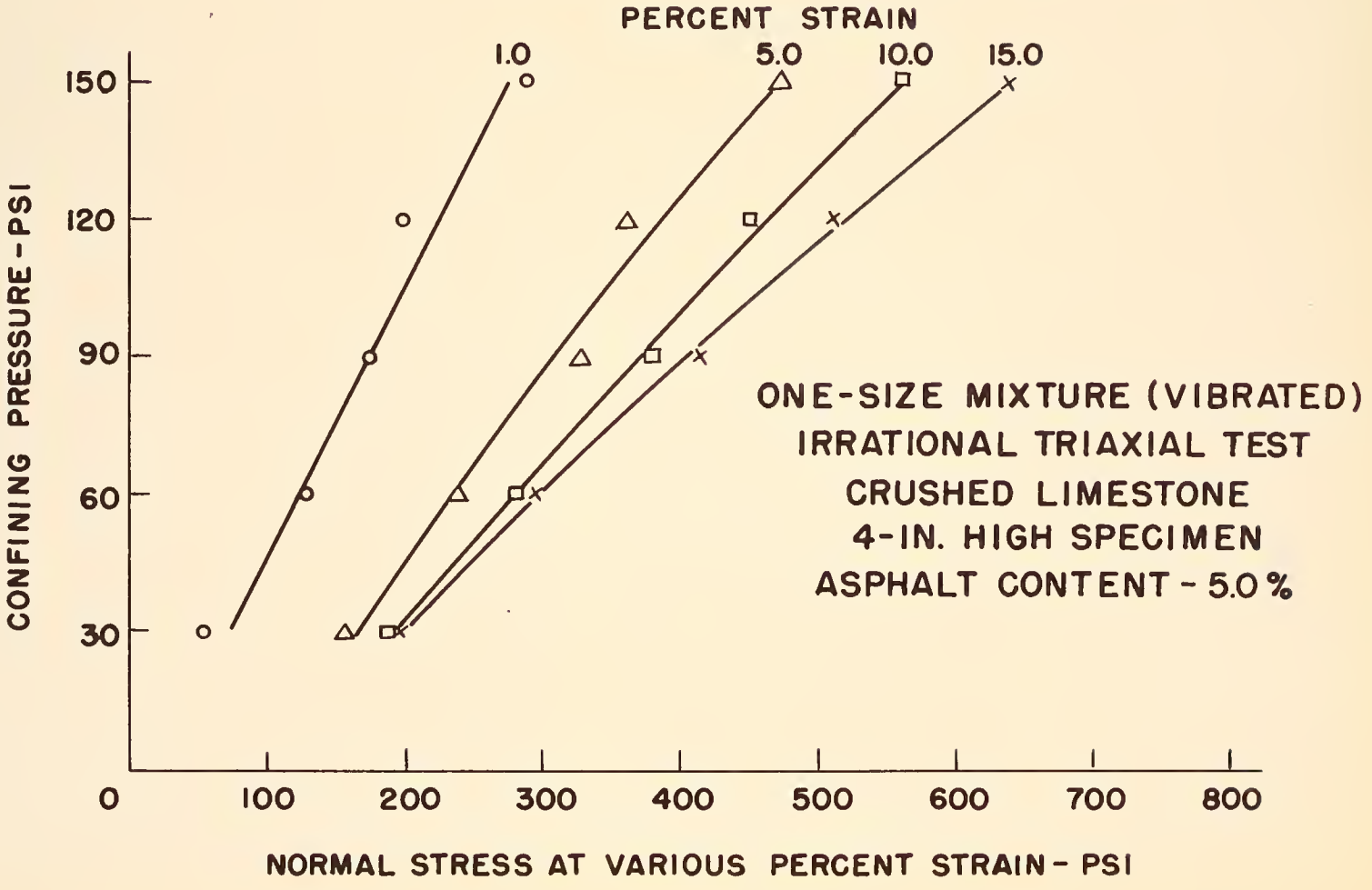


FIG. 15

strain is similar in each case to those representing the stability properties of the rational triaxial specimens. However, the irrational triaxial test data at 1.0 percent strain were best represented by a straight line for both degrees of compaction. At the three higher strain values of 5.0, 10.0, and 15.0 percent, the stress curves in both figures show that normal stress increased at an increasing rate as the confining pressure was increased.

A comparison of the rational test results as shown in Fig. 10 with the irrational test data in Fig. 14 for the non-vibrated specimens indicated that the normal stresses at 30 and 60 psi confining pressure were higher for the rational specimens (10 in. high) than for the irrational specimens (4 in. high), while the reverse was true for the higher confining pressures of 90, 120, and 150 psi. This phenomenon was previously noted in the study of the non-vibrated and vibrated specimens having a height of 10 in.

An appraisal of the data for the irrational test specimens for the two degrees of compaction brought out that greater shearing-strength values were determined for the more dense specimens (vibrated) as presented in Fig. 15 than for the less dense specimens (non-vibrated) as illustrated in Fig. 14, at all levels except the 1.0 percent strain value. The same pattern was evident when the rational (Fig. 13) and irrational (Fig. 15) test results for the vibrated specimens were compared. The increase in the lateral support of the 4-in. high specimens due to the shearing stresses developed between the ends of the specimen and the loading plates produced higher strength values in the irrational triaxial test at the 5.0, 10.0, and 15.0 percent strain. However, the

rational test specimens (vibrated) possessed greater triaxial stability than the irrational ones at a strain of 1.0 percent.

These discrepancies cannot be fully appraised as the stress properties of this one-size mixture are greatly influenced by the complicated interaction of lateral support, rearrangement of the aggregate particles, and reduction in the volume of the triaxial specimen during the test. The curvilinear pattern of these relationships between the confining pressure and the normal stress at various strains prevents the analysis of the stability properties in terms of such constant values as the cohesion and the angle of internal friction. Only the shearing stress of this mixture for any given condition of confining pressure and strain is a true representation of its strength.

SUMMARY OF RESULTS AND CONCLUSIONS

The following conclusions are drawn from the results of this laboratory investigation. However, these are to be considered valid only for the bituminous-aggregate mixtures of the same materials and gradings and the designated testing methods that were utilized in this study. Although field performance data are available for the mixture types used in this study, these conclusions have not been substantiated by field studies of the specific paving mixtures utilized, and their practical significance is limited in this respect.

1. The stress-strain curves representing the open-graded mixture were considerably different from those for the one-size mixture. The stress-strain diagrams for the open-graded mixture reached a peak normal stress value at a relatively low strain, while the normal stress continued to increase at a decreasing rate with an increase in the strain value for the one-size mixture.

2. When the open-graded mixture was evaluated by the rational and the irrational triaxial tests, the shearing strength developed was accurately predicted by the Coulomb equation and represented by the constant parameters, cohesion and angle of internal friction. However, the shearing strength values produced by the irrational test were higher than those obtained by the rational procedures because additional lateral support was afforded the short test specimens by the shearing stresses developed between the ends of the compacted specimens and the loading plates. Also, the slopes of these linear stress relationships of the rational and the irrational triaxial tests on the open-graded mixture were not the same, thus indicating that different values of cohesion and angle of internal friction were produced by these two testing procedures.

3. Low values of shearing stress were developed by the one-size mixture at low strain values and low confining pressures. However, when the specimen deformation and/or the confining pressure were increased, sufficient shearing resistance was developed to indicate more realistic stability values for this paving mixture. Thus, the strength of the one-size mixture may be evaluated satisfactorily in the laboratory if sufficient lateral support and the proper degree of deformation are provided to enable this bituminous-aggregate mixture to mobilize a significant portion of its potential shearing resistance. For the range of confining pressures considered, at least 90 percent of the available shearing resistance is developed if the compacted specimen is deformed to a strain of 10.0 percent. The conventional methods of triaxial compression testing as applied to bituminous mixtures must be modified to ascertain the complex stress-development pattern of this paving material.

4. Individual measures of the effects of specimen height and degree of compaction on the triaxial stability of the one-size mixture cannot be wholly isolated because of the interacting effects of lateral support, rearrangement of the aggregate particles, and change in specimen volume during test.

REFERENCES

1. Aldous, W. M., Herner, R. C., and Price, M. H., "The Load Transmission Test for Flexible Paving and Base Courses, Part II, Triaxial Test Data on Structural Properties of Granular Base Materials," Technical Development and Evaluation Center, Civil Aeronautics Administration, Technical Development Report No. 144, June, 1951.

2. American Society for Testing Materials, ASTM Standards, Part 3, Philadelphia, Pa., 1955.

3. Endersby, V. A., "The History and Theory of Triaxial Testing, and the Preparation of Realistic Test Specimens - A Report of the Triaxial Institute," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

4. Gibson, R. E., "Experimental Determination of the True Cohesion and True Angle of Internal Friction in Clays," Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, Session 2, Switzerland, August, 1953.

5. Goetz, W. H., and Chen, C. C., "Vacuum Triaxial Technique Applied to Bituminous-Aggregate Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 19, 1950.

6. Hennes, R. G., and Wong, C. C., "Physical Interpretation of Triaxial Test Data," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

7. Herrin, M., and Goetz, W. H., "Effect of Aggregate Shape on the Stability of Bituminous Mixes," Proceedings, Highway Research Board, Vol. 33, 1954.

8. Hveem, F. N., and Davis, H. E., "Some Concepts Concerning Triaxial Compression Testing of Asphaltic Paving Mixtures and Subgrade Materials," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

9. Lambe, T. W., Soil Testing for Engineers, John Wiley and Sons, Inc., New York, 1954.

10. McCarty, L. E., and Hank, R. J., "A Comparison of Dimensions in the Mohr Diagram with Those in Other Diagrams Used in Stress Analysis," Proceedings, The Association of Asphalt Paving Technologists, Vol. 18, 1949.

11. McLeod, N. W., "A Rational Approach to the Design of Bituminous Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 19, 1950.

12. McLeod, N. W., "Application of Triaxial Testing to the Design of Bituminous Pavements," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

13. McLeod, N. W., "Rational Design of Bituminous Paving Mixtures with Curved Mohr Envelopes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

14. McLeod, N. W., "The Design of Bituminous Mixtures with Curved Mohr Envelopes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

15. McLeod, N. W., "The Stability of Granular and Cohesive Materials in Triaxial Compression," Proceedings, The Association of Asphalt Paving Technologists, Vol. 17, 1948.

16. Monismith, C. L., and Vallerga, B. A., "Relationship Between Density and Stability of Asphalt Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 25, 1956.

17. State Highway Commission of Indiana, Standard Specifications for Road and Bridge Construction and Maintenance, 1952.

18. Stevens, D. E., "Fundamentals of Stability Testing of Asphalt Mixes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

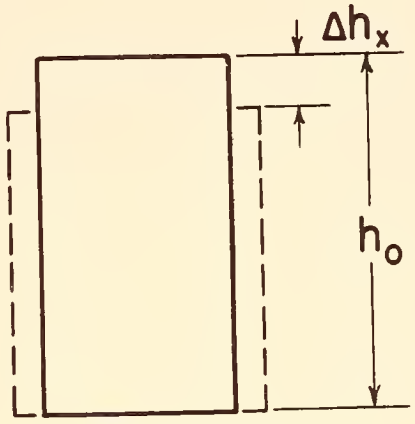
19. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1954.

20. The Asphalt Institute, Mix Design Methods for Hot-Mix Asphalt Paving, Manual Series No. 2, College Park, Md., 1956.

21. Oppenlander, J. C., "Triaxial Testing of Bituminous Mixtures at High Confining Pressures," a Thesis submitted to Purdue University for the degree of Master of Science in Civil Engineering, June, 1957.

APPENDIX A

DERIVATION OF RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING NO VOLUME CHANGE



ASSUMPTION: AT ALL TIMES THE TEST SPECIMEN IS CYLINDRICAL

WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA
Δh_x = CHANGE IN SPECIMEN HEIGHT
A_o = INITIAL CROSS-SECTIONAL AREA
h_o = INITIAL SPECIMEN HEIGHT
V_f = FINAL SPECIMEN VOLUME
V_o = INITIAL SPECIMEN VOLUME

DERIVATION: $V_f = A_x(h_o - \Delta h_x)$

$V_o = A_o h_o$

$V_o = V_f$ (NO VOLUME CHANGE)

$A_x(h_o - \Delta h_x) = A_o h_o$

$$A_x = \frac{A_o h_o}{(h_o - \Delta h_x)}$$

$$A_x = \frac{A_o}{\left(1 - \frac{\Delta h_x}{h_o}\right)}$$

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING NO VOLUME CHANGE

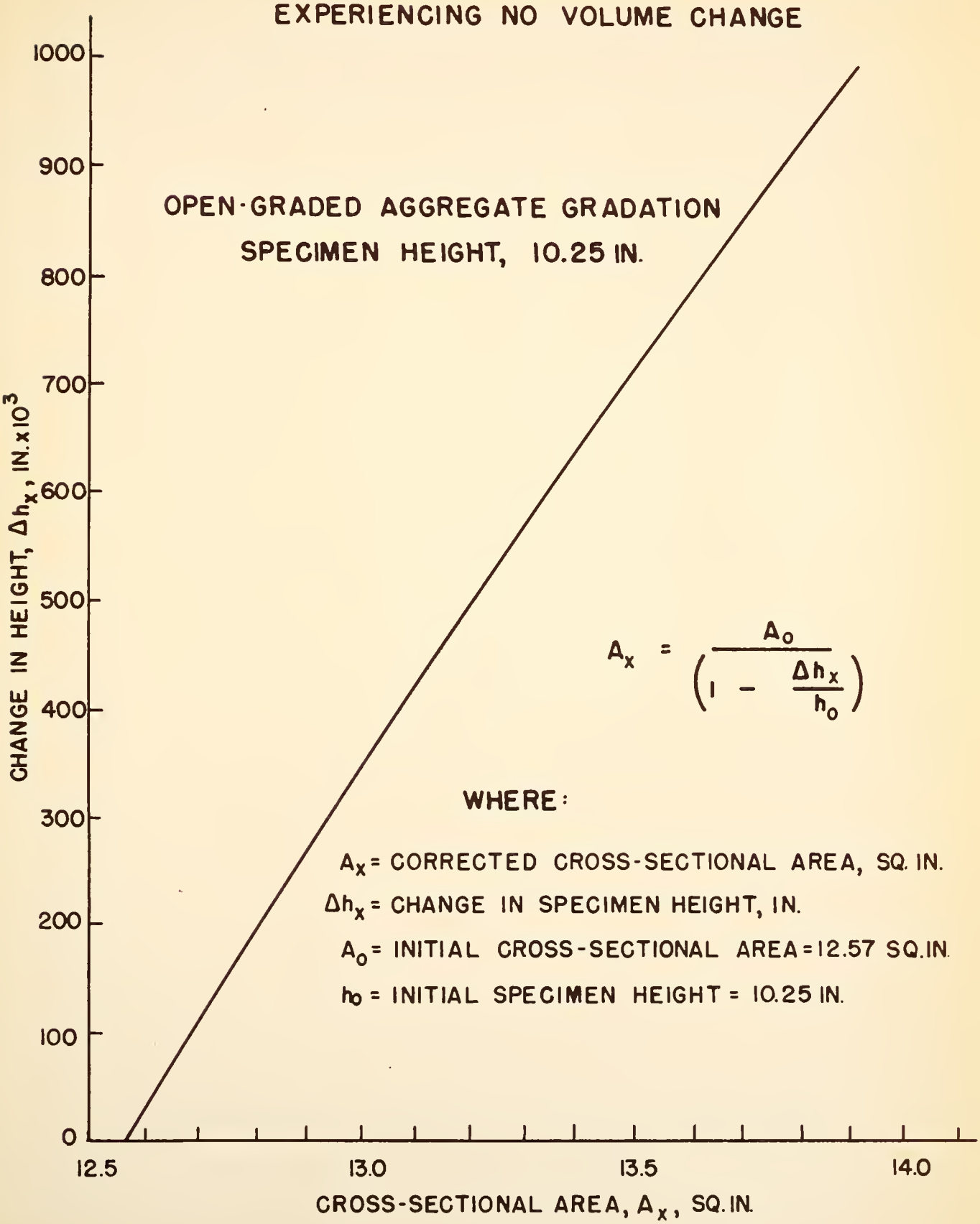
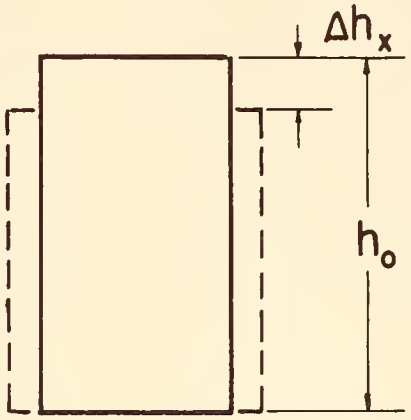


FIG. 16

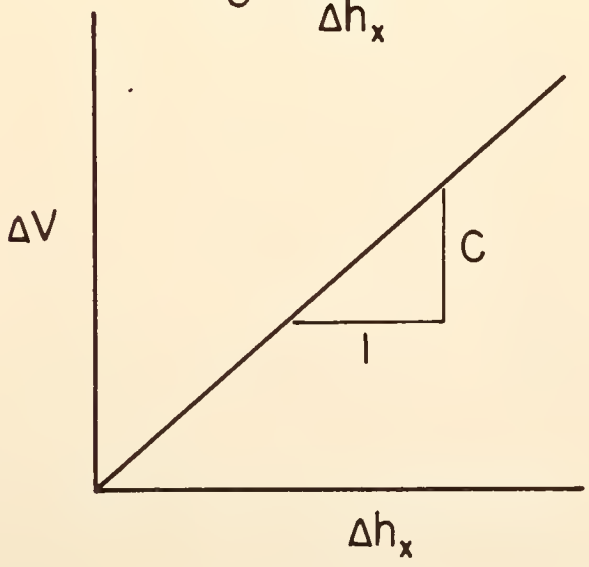
APPENDIX B

DERIVATION OF RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION



ASSUMPTION: AT ALL TIMES THE TEST SPECIMEN IS CYLINDRICAL

$$C = \frac{\Delta V}{\Delta h_x}$$



ASSUMPTION: THE CHANGE IN VOLUME IS PROPORTIONAL TO THE CHANGE IN HEIGHT

WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA

Δh_x = CHANGE IN SPECIMEN HEIGHT

A_o = INITIAL CROSS-SECTIONAL AREA

h_o = INITIAL SPECIMEN HEIGHT

V_f = FINAL SPECIMEN VOLUME

V_o = INITIAL SPECIMEN VOLUME

ΔV = CHANGE IN SPECIMEN VOLUME

C = VOLUME CHANGE PER CHANGE IN HEIGHT

DERIVATION:

$$V_f = A_x (h_o - \Delta h_x)$$

$$V_o = A_o h_o$$

$$C = \frac{\Delta V}{\Delta h_x}$$

$$V_f = V_o - \Delta V \quad \text{(VOLUME REDUCTION)}$$

$$A_x (h_o - \Delta h_x) = A_o h_o - C\Delta h_x$$

$$A_x = \left(\frac{A_o h_o}{h_o - \Delta h_x}\right)\left(1 - \frac{C\Delta h_x}{A_o h_o}\right)$$

$$A_x = \left(\frac{A_o}{1 - \frac{\Delta h_x}{h_o}}\right)\left(1 - \frac{C\Delta h_x}{A_o h_o}\right)$$

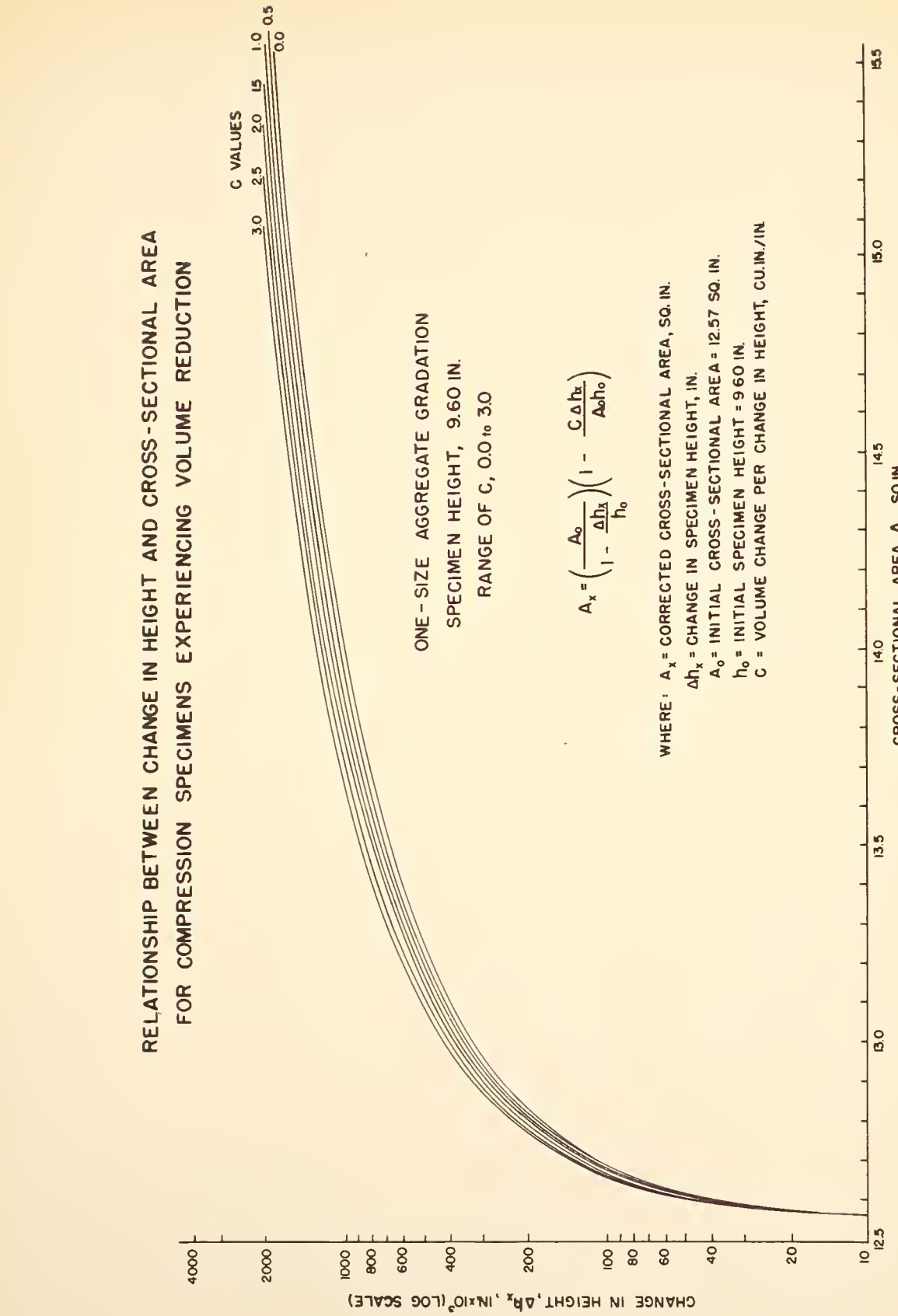

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA
FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION
C VALUES
0.0
0.5
1.0
1.5
2.0
2.5
3.0
ONE-SIZE AGGREGATE GRADATION
SPECIMEN HEIGHT, 9.60 IN.
RANGE OF C, 0.0 to 3.0
$A_x = \left(\frac{A_o}{1 - \frac{\Delta h_x}{h_o}}\right)\left(1 - \frac{C \Delta h_x}{A_o h_o}\right)$
WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA, SQ. IN.
Δh_x = CHANGE IN SPECIMEN HEIGHT, IN.
A_o = INITIAL CROSS-SECTIONAL AREA = 12.57 SQ. IN.
h_o = INITIAL SPECIMEN HEIGHT = 9.60 IN.
C = VOLUME CHANGE PER CHANGE IN HEIGHT, CU.IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.×10³ (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
13.0
13.5
14.0
14.5
15.0
15.5
CROSS-SECTIONAL AREA, A_x, SQ.IN.

FIG. 17

APPENDIX C

TABLE 4

Triaxial Test Results — Open-Graded Mixture

Crushed Limestone — 10-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi			Density lb/cu ft
	1.0% Strain	5.0% Strain	Peak	
0	---	---	164	148.2
0	---	---	168	146.8
0 (avg)	---	---	166	147.5
30	387	---	396	146.7
30	377	---	384	146.7
30 (avg)	382	---	390	146.7
60	431	---	445	147.4
60	434	---	455	147.3
60 (avg)	433	---	450	147.4
90	498	---	535	147.7
90	483	---	533	149.2
90 (avg)	491	---	534	148.5
120	492	586	590	146.7
120	512	594	594	146.3
120 (avg)	502	590	592	146.5
150	592	734	739	145.9
150	571	719	726	145.2
150 (avg)	582	727	733	145.6

TABLE 5

Triaxial Test Results — Open-Graded Mixture

Crushed Limestone — 4-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress psi	Density lb/cu ft
0	386	148.1
30	404	149.2
60	488	143.2
90	626	142.9
120	765	143.0
150	840	142.8

TABLE 5

Triaxial Test Results — One-Size Mixture

Crushed Limestone — 10-in. High Specimen

Asphalt Content = 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C*
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	$in.^3$ per in.
0	10	---	---	---	113.2	113.2	0.0	0.0
0	17	---	---	---	115.0	115.0	0.0	0.0
0 (avg)	14	---	---	---	114.1	114.1	0.0	0.0
30	99	141	154	158	112.9	113.4	0.5	1.20
30	106	143	154	156	111.7	112.2	0.5	1.26
30 (avg)	103	142	154	157	112.3	112.8	0.5	1.23
60	177	233	250	254	113.6	114.1	0.5	1.07
60	170	233	254	262	113.4	114.3	0.9	1.86
60 (avg)	174	233	252	258	113.5	114.2	0.7	1.47
90	226	282	314	324	113.8	115.2	1.4	0.98
90	223	274	290	332	113.4	114.2	0.8	0.60
90 (avg)	225	278	302	328	113.6	114.7	1.1	0.79
120	240	344	400	419	116.0	121.2	5.2	2.76
120	245	324	383	421	113.3	116.2	2.9	2.19
120 (avg)	243	334	392	420	114.7	118.7	4.0	2.48
150	296	429	511	540	115.3	117.2	1.9	1.46
150	310	414	497	536	115.1	120.9	5.8	6.10
150 (avg)	303	422	504	538	115.2	119.1	3.9	3.78

* C = Change in volume per change in height.

TABLE 7

Triaxial Test Results — One-Size Mixture

Crushed Limestone — 4-in. High Specimen

Asphalt Content = 5.0%

Confining Pressure psi	Normal Stress = psi				Density - lb/cu ft			C *
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	in³ per in
0	15	---	---	---	111.8	111.8	0.0	0.0
30	73	113	125	133	114.6	114.7	0.1	0.21
60	126	189	225	240	110.2	113.1	2.9	2.25
90	180	268	324	369	110.1	115.1	5.0	2.57
120	250	366	449	498	112.2	116.9	4.7	3.40
150	288	436	524	590	111.9	113.4	1.5	1.57

* C = Change in volume per change in height.

TABLE 8

Triaxial Test Results — One-Size Mixture (Vibrated)

Crushed Limestone — 10-in. High Specimen

Asphalt Content = 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C * in.^{3} per in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
30	88	119	131	136	115.5	116.8	1.3	1.02
60	160	213	231	235	115.6	115.6	0.0	0.01
90	194	282	330	357	118.6	119.0	0.4	0.47
120	259	359	420	464	117.3	121.1	3.8	3.68
150	325	448	523	557	116.7	119.2	2.5	2.22

* C = Change in volume per change in height.

TABLE 9

Triaxial Test Results — One-Size Mixture (Vibrated)

Crushed Limestone — 4-in. High Specimen

Asphalt Content = 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C *
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	$in.^3$ per in.
30	56	158	190	198	113.9	117.1	3.2	1.26
60	128	242	282	296	116.3	118.3	2.0	1.22
90	176	329	380	415	114.8	115.9	1.1	0.53
120	200	366	453	509	115.0	117.1	2.1	1.33
150	288	473	563	641	117.0	118.7	1.7	1.42

* C = Change in volume per change in height.